CHAPTER 3

METHODOLOGY, HYDROLOGIC, AND HYDRAULIC MODELING

3.1 Introduction

The purpose of this chapter is to report the results of an assessment made of limited hydrologic and hydraulic analysis performed on select master planned facilities identified by the City for this DMP Update.

The study locations identified by the City have been categorized into two types: (a) planned drainage system networks where development has occurred and current drainage facilities may be inadequate due to additional impervious areas, and (b) planned drainage system networks where development is scheduled to occur in the near future. Further, a discussion is presented on the results of hydraulic modeling which determines if the selected master planned facilities are necessary, in need of upgrade due to development or change in condition, or are in need of replacement. The planned facilities deemed necessary, were incorporated into the overall cost estimate for the PLDA fee funded projects, a topic discussed in Chapter 5.

3.2 Approach

Described below are assumptions and key watershed modeling parameters and values (derived from previous master plans, City GIS mapping, and other studies) that were used to create models that simulate the hydraulic conditions of the selected drainage systems.

3.2.1 Hydrology

Several studies, as well as the GIS information, contain mapping that identify storm water discharge locations and the drainage area associated with each discharge location. Defining the drainage areas contributing storm water runoff to the drainage system is an important aspect in determining contributing sources and in accurate assessment of the capacity of the existing or proposed drainage system. Based on the existing topography and GIS information, drainage area parameters, such as slopes, land use and imperviousness, can be determined. After the drainage area boundaries have been identified, the discharge from each drainage area can be calculated using the Storm Water Management Model, Version 5 (SWMM 5.0) [model selection is further discussed in a later section].

SWMM 5.0 is a widely-used computer program that allows flexibility for modeling the quantity of storm water runoff from basins and drainage systems. This program was developed for the United States Environmental Protection Agency and is approved by the Federal Emergency Management Agency (FEMA) for use in flood insurance studies. SWMM 5.0 contains several different hydrologic/hydraulic simulation modules and also includes data management modules that provide the framework for its flexibility.
The EPA SWMM 5.0 is a dynamic rainfall-runoff simulation model used for single event or long-term (continuous) simulation of runoff quantity and quality from primarily urban areas. Running under the Windows format, SWMM 5.0 provides an integrated environment for editing study area

input data, running hydrologic, hydraulic and water quality simulations, and viewing the results in a variety of formats. These include color-coded drainage area and conveyance system maps, time series graphs and tables, profile plots, and statistical frequency analyses (USEPA, 2005).

The preparation of a base hydrology model began with the collection of standard and assumed parameters. These were then incorporated into a standard (base model) drainage system conveyance network within the SWMM 5.0 software package. After the base model was developed and an appropriate design storm selected, a hydrologic simulation was generated. The output of the simulation was reviewed and compared to standard Rational Method hand calculations to determine difference/percent error between methods.

The San Diego County Hydrology Manual, prepared by the County of San Diego Department of Public Works Flood Control Section (June 2003), specifies three methods for assessing hydrology: the Rational Method, the Modified Rational method, and the Natural Resources Conservation Service (NRCS) Hydrologic Method. For this study, the Rational Method was used for drainage areas smaller than one square mile. The Modified Rational Method was used for drainage areas smaller than one square mile where more than one drainage area discharges to a drainage system. Finally, the Natural Resources Conservation Service (NRCS) Hydrologic Method was used for drainage areas greater than 1 square mile. The Rational Method and Modified Rational Method require determination of the drainage area (A), a storm water runoff coefficient (C) of existing surface materials, and historical rainfall intensity (I) for the area. These methods were utilized to provide a check of the results from the SWMM 5.0 program output.

The value for C is dependent on the type of surface material. The San Diego County Hydrology Manual contains C values for different surface materials. A weighted C value was developed for each drainage area using the storm water runoff coefficient assigned to each surface material, the total area for each surface material, and the total area of each drainage area.

The San Diego County Hydrology Manual includes a procedure for determining the value for rainfall intensity **I**. The 6-hour and 24-hour precipitation from a county wide isopluvial map was used to create an Intensity-Duration curve that can determine the rainfall intensity. The San Diego County Hydrology Manual also specifies the use of a 50-year design storm frequency for drainage facilities upstream of major roadways, and a 100-year design storm frequency for all facilities at major roadways. The more stringent requirements of the San Diego County Hydrology Manual were typically used.

The flow rate, **Q**, from the drainage area into the drainage system was determined using the following relationship:

$$Q = kCIA$$

where	Q	= Flow rate, cubic feet per second
	C	= Storm water runoff coefficient, dimensionless
	k	= Conversion factor, feet*day/second*inch
	I	= Historical rainfall intensity, inch per day
	A	= Surface area of drainage basin, square feet

The flow rate, **Q**, was used to assess the capacity of the drainage system and to determine the storage volume and required retention time for existing sedimentation basins, where appropriate.

3.2.2 Hydraulics

The existing and proposed drainage systems within the City of Carlsbad typically consist of catch basins, junction structures, manholes, underground conveyance system, open channels, and associated outfalls. For this investigation, an outfall is identified as the point at which storm water is discharged from the drainage system into the receiving water body, to an adjacent property, or to an adjacent drainage system. The drainage area boundary for each outfall forms a distinct sub-basin that collects surface water runoff. The boundaries of several drainage areas extend beyond the City of Carlsbad property boundary and collect storm water runoff from the adjacent properties.

Drainage systems must adequately convey peak storm water flows during its design life. Peak flows anticipated for each drainage basin were determined and the ability for the pertinent, existing drainage system to convey the peak flows evaluated. Flows contributed by future connections were also considered when evaluating existing systems. Evaluation of the drainage system requires detailed maps of the drainage areas, catch basins and other appurtenances, storm drains, underground conveyance systems, and open channels. Information required for hydraulic assessment of the drainage systems includes the following:

- the dimension and shape of the conveyance structures;
- the slope of the conveyance structures;
- the conveyance structure materials;
- the storm drain inlet type and dimensions;
- the surface or finished grade elevations; and
- the flow line or invert elevations and flow capacity of systems.

3.3 Model Selection

The City has sponsored and/or authorized numerous hydrology/hydraulic studies that have utilized various types of proprietary, as well as public domain software, to determine infrastructure capacity and runoff quantities from their facilities. In an effort to provide a reliable model that is widely accepted by outside agencies, readily available, and easy to understand, a criteria for usage was established to determine the best fit model for the City. The general criteria used for selection include:

- availability;
- acceptance by other entities;
- use of general drainage parameters;
- organized input and output;
- ease of manipulation;
- availability of end user manual;
- ability to export to other programs; and
- generation of consistent and reliable results.

Based on the above selection criteria, the best fit model for the task of storm water modeling for the DMP Update is the Storm Water Management Model, version 5 (SWMM 5.0). The SWMM 5.0 model is readily available from the EPA web page; it is a widely-used computer program that is accepted by other agencies; allows flexibility for modeling the quantity of storm water runoff from basins and drainage systems; and can be easily manipulated and exported to other programs. There is a manual available with tutorials that provided online training for model development.

3.3.1 Modeling Capabilities

The SWMM 5.0 model has two major components that serve to calculate stormwater volumes: the RUNOFF Module and the TRANSPORT module. The RUNOFF module of SWMM 5.0 utilizes the precipitation that falls on sub-watersheds and calculates the corresponding discharge (**Q**), based on area (**A**), calculated rainfall intensity (**I**) and computed runoff coefficient. The TRANSPORT module routes the calculated flow (**Q**) through a corresponding drainage system that can be composed of pipes, channels, storage/treatment devices, pumps, and regulators as required. The model tracks the quantity and quality of runoff generated from each sub-watershed as well as computing the flow rate, flow depth, and quality of water in each drainage component during the simulation. SWMM 5.0 accounts for various hydrologic processes that produce runoff from urban areas, including:

- time-varying rainfall
- evaporation of standing surface water
- snow accumulation and melting
- rainfall interception from depression storage
- infiltration of rainfall into unsaturated soil layers
- percolation of infiltrated water into groundwater layers
- interflow between groundwater and the drainage system
- nonlinear reservoir routing of overland flow

Spatial variability in all of these processes is achieved by dividing a study area into a collection of smaller, homogeneous sub-catchment areas, each containing its own fraction of pervious and impervious sub-areas. Overland flow can be routed through sub-areas, sub-catchments, or entry points of a drainage system. SWMM 5.0 also contains a flexible set of hydraulic modeling capabilities used to route runoff and external inflows through the drainage system network of pipes, channels, storage/treatment units, and diversion structures. These allow the model to

- handle networks of unlimited size;
- use a wide variety of standard closed and open conduit shapes as well as natural channels;
- model special elements such as storage/treatment units, flow dividers, pumps, weirs, and orifices;
- apply external flows and water quality inputs from surface runoff, groundwater interflow, rainfall-dependent infiltration/inflow, dry weather sanitary flow, and user-defined inflows;
- utilize either kinematic wave or full dynamic wave flow routing methods;
- model various flow regimes, such as backwater, surcharging, reverse flow, and surface ponding; and
- apply user-defined dynamic control rules to simulate the operation of pumps, orifice openings, and weir crest levels

In addition to modeling the generation and transport of runoff flows, SWMM 5.0 can also estimate the production of pollutant loads associated with this runoff. The following processes can be modeled for any number of user-defined water quality constituents:

- Dry-weather pollutant buildup over different land uses;
- Pollutant washoff from specific land uses during storm events;
- Direct contribution of rainfall deposition;
- Reduction in dry-weather buildup due to street cleaning;
- Reduction in washoff load due to BMPs;
- Entry of dry weather sanitary flows and user-specified external inflows at any point in the drainage system.

3.3.2 Typical Applications of SWMM 5.0

Since its original development, in 1971, the SWMM 5.0 program package has been used in thousands of sewer and stormwater studies. The current version has robust features to handle typical applications such as:

- Design and sizing of drainage system components for flood control;
- Sizing of detention facilities and their appurtenances for flood control and water quality protection;
- Flood plain mapping of natural channel systems;
- Designing control strategies for minimizing combined sewer overflows;
- Evaluating the impact of inflow and infiltration on sanitary sewer overflows;
- Generating non-point source pollutant loadings for waste load allocation studies;
- Evaluating the effectiveness of BMPs for reducing wet weather pollutant loadings.

For the DMP Update, SWMM 5.0 was used to determine the runoff quantity and flow rates for the selected projects.

3.4 Hydrologic and Hydraulic Design Criteria

Described in the following section are the components necessary to determine the hydrology of contributing basins within the geographic area of the City. The hydrologic components provide the backbone for the hydraulic methods used to calculate and determine the capacity of proposed drainage facilities within the city limits. Model parameters include, but are not limited, to rain gauge information, sub-watershed delineation (area, slope, land use, Manning's roughness coefficient **n**), runoff parameters, conveyance system, and modeling scenarios.

3.4.1 Rain Gauge Information

Rain gauges located in the City have been in service since 1959. Although extensive, it is insufficient in determining a 100-year event. A method to develop a synthetic rainfall distribution is presented in the San Diego Hydrology Manual, allowing prediction of long-term events (such as a 100-year storm) using smaller data sets. The method is from the Soil Conservation Service (SCS) and is commonly known as the Type B storm distribution. The Type B was developed to represent a normalized rainfall event for various storm frequencies that occur in the western portion of the San Diego County based on historical rainfall data (San Diego County, 2001).

3.4.2 Watershed/Sub-Watershed Delineation

The determination of contributing drainage area is based on topography (topographic divide that identifies high point and direction of flow), the site conditions (developed or open space), and the existing or proposed drainage conveyance. The GIS mapping provided by the City includes 2-foot contours, delineation of developed areas such as roadways, city streets and other defining geographic features that allow determination of drainage watersheds and components. A watershed is a land mass that funnels runoff towards a waterway such as a creek, river or lake. Components that affect the watershed model are impervious areas, natural conveyances and the location of existing or proposed drainage system features. Other man-made features such as development that changes the topography or detention basins within the watershed have been reviewed and taken into account. Ultimate build-out was assumed and modeled as the worse case generator of storm water runoff within the watersheds. It is noted that the terms "watershed" and "catchment" have been used interchangeably within the context of this document.

3.4.3 Infiltration/Percent Runoff

Other factors that affect watershed and sub-watershed calculations are the ability to retain water within a soil matrix or soil type. This retention of water is created by the initial infiltration of runoff into the ground at the beginning of a rainfall event. The infiltration rate is directly correlated to the soil type. From soil mechanics, it is known that soil particles with large diameters, coupled with low cohesion properties (e.g., loose sand), provide greater infiltration. There will be less runoff from sub-watersheds that have greater infiltration (i.e. pervious) capacity. This infiltration capacity is also related to the land use categories provided by the City.

The San Diego Association of Governments (SANDAG) provides hydrologic soils type and coverage for the City of Carlsbad. From this mapping, based on the USDA Soil Survey Handbook for the San Diego area, it can be seen that the predominant soil type found within the City is Type D. This soil (Type D) is made up mostly of clays, resulting in low infiltration rates and increased surface runoff. Twelve percent of the open area soils in the City are Type C, a soil that consists mostly of fine silts. Six percent of the open areas in the City, mostly inland, have Type B soils that consist of silts and/or fine sands. Type A soils are made up of coarser draining sand and can be found in the coastal areas, making up to eleven percent of the open area soils found within the City.

Having determined the soil type, infiltration parameters were selected for model input. Infiltration in SWMM 5.0 can be calculated using any of the three methods: Horton, Green-Ampt, or Curve Number. The Green-Ampt Equation was selected for determining infiltration rates from typical soil texture classifications. This method makes use of predicted parameters that reflect various soil types. Average values are presented in Table 3.4-1 for each soil texture class. To better reflect the saturated soil conditions that produce the most runoff and therefore a worse case scenario, the initial moisture deficit was assumed to be equal to zero. Since most of the sub-watersheds contained multiple soil types, a weighted average of the infiltration parameters was calculated.

Table 3.4 –1
GREEN-AMPT SOIL PARAMETERS

Soil Texture Class	Hydraulic Conductivity (in/hr) K	Suction Head (in) Ψ	Porosity φ	Field Capacity FC	Wilting Point WP
Sand	4.74	1.93	0.437	0.062	0.024
Loamy Sand	1.18	2.40	0.437	0.105	0.047
Sandy Loam	0.43	4.33	0.453	0.190	0.085
Loam	0.13	3.50	0.463	0.232	0.116
Silt Loam	0.26	6.69	0.501	0.284	0.135
Sandy Clay Loam	0.06	8.66	0.398	0.244	0.136
Clay Loam	0.04	8.27	0.464	0.310	0.187
Silty Clay Loam	0.04	10.63	0.471	0.342	0.210
Sandy Clay	0.02	9.45	0.430	0.321	0.221
Silty Clay	0.02	11.42	0.479	0.371	0.251
Clay	0.01	12.60	0.475	0.378	0.265

(Rawls, 1983)

The City also has GIS shape files that map land uses categories. This information is captured and has been updated in the 2004 Carlsbad General Plan (General Plan). The General Plan Land Use (GPLU) categories developed as part of the Carlsbad General Plan is a distribution of land uses that represents the desirable pattern for ultimate build out and takes into consideration future development. It is noted that the land use categories may be altered as the City evolves. However, the intent of the General Plan is to ensure that an adequate level of public facilities will be provided at all times.

A correlation between percent impervious cover and GPLU category were derived in the Center for Water Protections (CWP), Rapid Watershed Planning Handbook, (CWP, 1998). This correlation provides the percent impervious cover for the designated land use categories that can be found in typical city arrangements. This percent impervious cover was incorporated into the model as part of the sub-watershed properties to determine runoff coefficients. In addition, this value will play a role in determining runoff loading requirements for PLDA parameters that will be discussed in Chapter 5. Table 3.4-2 shows the assigned GPLU codes with land use descriptions and associated impervious cover in percent.

Table 3.4-2
IMPERVIOUSNESS BY GENERAL PLAN LAND USE

GPLU Code	Land Use Description	Impervious Cover (%)	Pervious Cover (%)
C/O/PI	Community Commercial/Office & Related Commercial/Planned Industrial	85	15
C/O/RMH	Community Commercial/Office & Related Commercial/Medium-High Density	85	15
E	Elementary School	55	45
G	Governmental Facilities	85	15
C/O	Community Commercial/Office & Related Commercial	95	15
H	High School	55	45
HC	Continuation School	55	45
J	Junior High School	55	45
N	Neighborhood Commercial	85	15
O	Office & Related Commercial	85	15
OS	Open Space	3	97
P	Private School	55	45
PI	Planned Industrial	85	15
PI/O	Planned Industrial/Office & Related Commercial	85	15
R	Regional Commercial	85	15
RH	High Density Residential	40	60
RH/C/O	High Density Residential/Community Commercial/Office and Related Commercial	70	30
RH/O	High Density Residential/Office & Related Commercial	70	30
RL	Low Density Residential	20	80
RLM	Low-Medium Density Residential	25	75
RM	Medium Density Residential	30	70
RM/O	Medium Density Residential/Office & Related Commercial	38	62
RMH	Medium-High Density Residential	40	60
RMH/O	Medium-High Density Residential/Office & Related Commercial	50	50
RMH/T-R	Medium-High Density Residential/Travel/Recreation Commercial	40	60
T-R	Travel/Recreation Commercial	85	15
T-R/C	Travel/Recreation Commercial/Community Commercial	85	15
T-R/O	Travel/Recreation Commercial/Office & Related Commercial	85	15
T-R/O/OS	Travel/Recreation Commercial/Office & Related Commercial/Open Space	80	20
TC	Transportation Corridor	100	0
U	Public Utilities	50	50
UA	Unplanned Areas	3	97
V	Village	85	15

(Derived from the *Center for Water Protections (CWP), Rapid Watershed Planning Handbook,* (CWP, 1998))

To ensure that the impervious percentages accurately reflected the existing mapped portions of the City, a comparison between Table 3.4-2 and the latest aerial photograph was performed. The latest aerial photograph of the City that was available at the time of this study was taken in 2001. For developed areas that did not fall within the estimated "norms" of Table 3.4-2, or required a more robust determination, the impervious cover was estimated based on paved areas contained within the sub-watershed. The ratio of the impervious area divided by the total area provides the percent impervious of the sub-watershed. The following formula was used to determine the runoff coefficient **C**, based on the percent imperviousness and soil type.

$$*\mathbf{C} = 0.90 \times (\%\ \text{Impervious}) + \mathbf{C_p} \times (1 - \%\ \text{Impervious})$$

where $\mathbf{C_p}$ =Pervious Coefficient Runoff Value for the soil type.

*San Diego County Hydrology Manual, June 2003.

3.4.4 Other Sub-watershed Parameters

SWMM 5.0 requires a value be entered for a "*Width*" parameter that is related to the time of concentration. This parameter is typically estimated by dividing the total sub-watershed area by its maximum length of overland flow. The GIS mapping was utilized to calculate the contributing area of flow and the maximum length of travel for each sub-watershed. The "*Width*" was then calculated based on the area and length of travel.

SWMM 5.0 also requires a value be entered for a "*% Slope*" parameter that is related to the average slope along the various pathways of overland flow. This value is used for the portions of the sub-watershed in which runoff is not confined within a channel or pipe.

3.4.5 Manning's Roughness Coefficient

The Manning's roughness coefficient "**n**" (Manning's n) for the sub-watershed pervious and impervious areas represents the roughness characteristics of the surface and is influenced by vegetation, channel irregularities, channel alignment, and scouring. Values of Manning's n are not as well defined for overland flow because of the considerable variability in ground cover, transitions between laminar and turbulent flow, obstructions encountered through the flow path, very small depths, etc. Typical values of Manning's n are listed in Table 3.4-3 and Table 3.4-4 respectively. Assumed values that were utilized for modeling purposed are given where available.

Table 3.4-3
MANNING'S "N" VALUES FOR VARIOUS PIPE MATERIALS

Conduit Material	Typical Manning's Roughness Coefficient "n"	Assumed Manning's Roughness Coefficient "n"
Asbestos-cement pipe	0.011 – 0.015	0.013
Brick	0.013 – 0.017	0.015
Cast iron pipe		
- Cement-lined & sealed coated	0.011 – 0.015	0.013
Concrete (monolithic)		
- Smooth forms	0.012 – 0.014	0.013
- Rough forms	0.015 - 0.017	0.016
Concrete pipe	0.011 – 0.015	0.013
Corrugated-metal pipe (1/2-in. x 2-2/3-in corrugations)		
- Plain	0.022 – 0.026	0.024
- Paved invert	0.018 – 0.022	0.020
- Spun asphalt lined	0.011 – 0.015	0.013
Plastic pipe (smooth)	0.011 – 0.015	0.013
Vitrified clay		
- Pipes	0.011 – 0.015	0.013
- Liner plates	0.013 – 0.017	0.015

Table 3.4-4
MANNING'S "N" VALUES FOR CHANNELS

Channel Type	Typical Manning's Roughness Coefficient "n"	Assumed Manning's Roughness Coefficient "n"
Lined Channels		
- Asphalt	0.013 – 0.017	0.015
- Brick	0.012 – 0.018	0.015
- Concrete	0.011 – 0.020	0.016
- Rubble or riprap	0.020 – 0.035	0.028
- Vegetal	0.030 – 0.40	0.035
Excavated or dredged		
- Earth, straight and uniform	0.020 – 0.030	0.025
- Earth, winding, fairly uniform	0.025 – 0.040	0.032
- Rock	0.030 – 0.045	0.038
- Unmaintained	0.050 – 0.140	0.100
Natural channels (minor streams, top width at flood stage < 100 ft.)		
- Fairly regular section	0.030 – 0.070	0.050
- Irregular section with pools	0.040 – 0.100	0.080

3.5 Design Methodology

The preparation of the model began with the collection of standard and assumed parameters. These were then incorporated into a standard (base model) drainage system conveyance network within the SWMM 5.0 software package. After the base model was developed and an appropriate design storm selected, a simulation was generated. A discussion on the modeling process and a brief overview of how SWMM 5.0 utilized the information to determine the runoff produced by a sub-watershed are presented in the following section.

3.5.1 Design Storm

As discussed in a previous section, a method to develop a synthetic rainfall distribution is presented in the San Diego Hydrology Manual, allowing prediction of long-term events (such as a 100-year storm) using smaller data sets. The method is from the SCS and is commonly known as the Type B storm distribution. The Type B was developed to represent a normalized rainfall event for various storm frequencies that occur in the western portion of the San Diego County based on historical rainfall data (San Diego County, 2001).

A continuous simulation model was considered to be more accurate, since it can better approximate runoff peaks and volumes. However, real storm data is required for continuous simulations. There are no rain gages within the City boundary that have a long enough record to statistically represent the required 100-year storm of interest. Thus, based on traditional rational methods identified in the San Diego County Hydrology Manual the design storm chosen for modeling purposes is the 100-year, 6 hour event. Selection of this storm frequency is to provide reasonable control of stormwater runoff from impervious surfaces and to satisfy the City of Carlsbad design standards.

3.5.2 Model Assumptions

The SWMM 5.0 model has several special parameters that manipulate runoff characteristics from a watershed during the calculation process. These parameters try to capture the initial occurrences that are created during the typical rainfall event. Careful consideration must be given to the assigned values for "*Depression Storage*", "*Zero Percent Detention*" and "*Initial Moisture Deficit*". Although these concepts are based on experimentation and empirical observations, their rudimentary computations reflect field conditions that closely match laboratory data results. The model parameters and assumptions made to determine the assigned values are discussed in this section.

The first special SWMM 5.0 parameter that relates to runoff routing is "*Depression Storage*", which is the loss or "initial abstraction" caused by such phenomena as surface ponding, surface wetting, interception and evaporation. For example, a backyard lawn is not always completely flat, but has many little depression pockets (*Depression Storage*) where water is stored. The depression storage volume, in inches [mm], must be filled prior to the occurrence of runoff. Depression storage may be treated as a calibration parameter, particularly to adjust runoff volumes. Separate depression storage values are required for pervious and impervious areas. Examples of typical "*Depression Storage*" values are given in Table 3.5-1.

Table 3.5-1
TYPICAL DEPRESSION STORAGE VALUES

Impervious surfaces	0.05 – 0.10 inches
Lawns	0.10 – 0.20
Pasture	0.20 inches
Forest litter	0.30 inches

(ASCE, 1992)

Another SWMM 5.0 parameter that relates to runoff routing is the "*Zero Percent Detention*". This parameter represents the impervious area percentage of the sub-watershed that can be set to have no detention storage, thus, creating immediate runoff. This parameter assigns a percentage of runoff (0.0-100.0 percent) from the impervious area in order to promote immediate runoff. For all the models, this parameter was set to 100 percent so that runoff would be instantaneous.

SWMM 5.0 also requires a parameter that describes the infiltration of precipitation into the soil at the beginning of the storm event. The "*Initial Moisture Deficit*" is the fraction difference between soil porosity and actual moisture content; it is non-dimensional. Initial moisture deficit is determined by the soil type. Values for dry, antecedent conditions tend to be higher for sandy soils than clay soils because the water is held weakly in the soil pores of sandy soils. However, for the models created as part of this study, the value was set to zero. An "*Initial Moisture Deficit*" of zero means that the soil porosity it equal to the actual moisture content, thus, the soil is saturated. A saturated soil will promote immediate runoff creating higher runoff volumes depicting the worse case scenario during a rainfall event.

Other modeling assumptions dealt with how the runoff generated from the surface characteristics would flow over the watershed. Two-foot contours from the City's GIS mapping information were used to determine elevation differences (high and low points) of the watershed. The change in elevation was then divided by the longest flow path. This, in turn, determined the speed at which the runoff would travel over the watershed as a function of time. For modeling purposes, it was assumed that the storm water runoff was contained within city streets. In addition, overland flow was directed to the drainage inlets and conveyance systems by following gutters located along the edges of the city streets.

3.5.3 Model Creation

Once all of the key parameters were selected, they were entered into the SWMM 5.0 modeling program. The primary watershed information such as land area, width, and slopes were entered to define the watersheds. Next, all input parameters discussed in previous sections were entered, including designating outlet points, storm water inlets, or simply a point to which the watershed drains. Each watershed was assigned a rain gauge to characterize a design storm time series; the design storm depends on the selected return period. The rain gauge is created by selecting the desired storm event and creating a rainfall distribution table over an assumed storm duration for the event.

3.5.4 Surface Runoff

Stormwater runoff that is generated from upstream sub-watersheds flows to the low point, along with the accumulating precipitation. This collection of water is considered the "Inflow" to a watershed. The "Outflow" of a watershed includes infiltration, evaporation, and surface runoff. The maximum *depression storage* ($\mathbf{d_p}$), discussed previously, is the capacity of the "reservoir." The surface runoff (**Q**) occurs when the depth of water in the "reservoir" exceeds the $\mathbf{d_p}$. When this occurs, the outflow is given by Manning's equation.

$$\mathbf{Q = (1.49/n)(\mathit{AR}^{2/3}\ \mathit{S}^{1/2})}$$

where:

Q = discharge, in cubic feet per second
A = cross-sectional area of flow, in square feet
R = hydraulic radius, A/WP, in feet
WP = wetted perimeter of flow, in feet
S = slope of the energy gradient
n = Manning's roughness coefficient

A water balance equation over the sub-watershed is solved numerically, continuously updating the depth of water (**d**) over the sub-watershed (USEPA, 2005). SWMM 5.0 treats surface runoff from a watershed as a nonlinear reservoir. The conceptual depiction of what occurs on the watershed surface is shown on Figure 3-5.

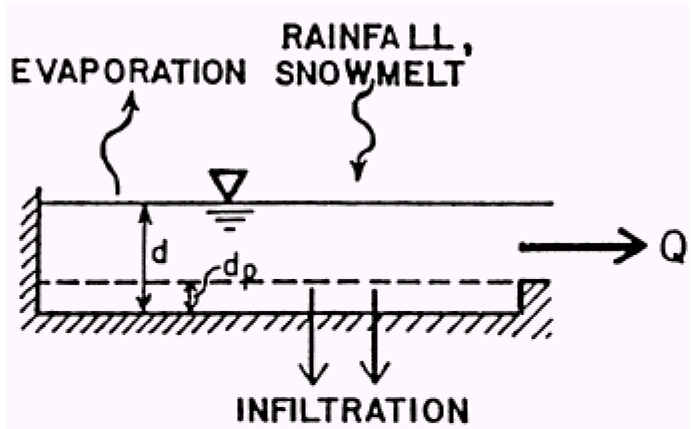


Figure 3-5. Conceptual View of SWMM 5.0 Surface Runoff

3.5.5 Conveyance System

The City's conveyance system consists mainly of drainage inlets for interception of runoff, junction structures that connect inlets and pipe networks, culverts and channels that transport runoff. Most of these facilities have been located, described, and incorporated into the existing storm drain system inventory that has been prepared and is contained within the City GIS mapping. "SD_Pipe" and "SD_Structures" are GIS shape files provided by the City that provide the inventory information of the specific City infrastructure. These files contain information such as: pipe/culvert size, slope, material type, and elevations based upon the city geographic coordinate system. For purposes of this study, the existing infrastructure will be used to connect new or proposed project features as required by the hydraulic modeling.

The conveyance system model consists of two basic elements: nodes and links. Nodes are points that connect conveyance links together such as junctions that can be made up of manholes and inlets, storage units representing detention basins, and flow dividers that divert overflow or convey runoff over a weir structure. An outfall is considered a special node as they represent the endpoint of a series of links and nodes. Nodes also represent free outfalls or connections to existing drainage features. Links are the conveyance components of a drainage system that are between a pair of nodes. Links can be conduits such as culverts or channels, pumps to lift water to a higher elevation, and/or regulators that are used to control and divert flow within a conveyance system.

Models developed for this study will employ the use of nodes such as junctions, and storage units as well as links which are pipe conduits, and open channels. Elevations for each particular component were determined from available GIS mapping or other various sources as described in the section Data Assumptions.

3.5.6 Data Assumptions

The typical drainage system requires and entrance and exit (nodes and links) as well as their respective parameters such as elevations, slopes, etc. However, not all information was readily available from the GIS attribute files as discussed earlier. Aerial photographs and topographic maps with two-foot contours were also utilized to determine ground line elevations for input into the model. City design standards were used as a guide to determine typical roadway widths, slopes and curb and gutter geometry. Where appropriate, the San Diego Regional Standard Drawings were utilized to provide a means for determining appropriate sizing of junctions and depth of cover for assumed pipe networks. The conveyance systems developed for this project as well as the assumptions made for the completion of a pipe network are discussed below.

The main components that describe nodes are the invert elevation, **e**, and the height or depth from invert to ground surface, **h**. The City GIS attribute files provided some of this critical information at existing drainage locations. Other information was collected from approved developer drawings, where available. When drainage component information was missing, the following assumptions were made regarding the type of manhole, cleanout or inlet that was necessary to complete a drainage system. If a junction was proposed, but there is no attribute information, the junction structure was assumed to be a Type A or Type B cleanout for modeling purposes. Type A cleanouts were used for conditions where inlet and outlet pipes had diameters less than or equal to 30 inches. If the diameter of any inlet or outlet pipe was 36 inches or greater, a Type B cleanout was selected. The San Diego Regional Standard Drawings were utilized to provide standard structures, inlet configurations and appropriate depths of fill over the pipe network. The assumption of four feet of cover as the depth of fill over culverts, and usage of standard structures would results in a uniform cost estimate that is dependent on depths and footprint of excavation.

Links are the conveyance component of the model that comprise of overland flow, pipes, culverts, concrete channels, and natural channels. All pipe networks were assumed to have a 4-foot depth of cover unless specific information was found describing the actual cover depth. Culvert slopes were determined by the conduit lengths and the difference in elevation between the upstream and downstream nodes. All pipes and culverts were modeled as circular channels. Sheet flow over a city street was modeled as an open rectangular channel with variable width and height depending on the upstream inflow. All culvert diameter information, concrete and natural channel dimensions were extracted from the information provided in the City GIS attribute files. Where information was not available, channel geometry was estimated based on aerial mapping.

3.5.7 Flow Routing

Once the physical geometry and details of the conveyance system has been entered, the appropriate routing method must be selected so that the simulation of flow can be performed through the conduits. Flow routing through a pipe network employs the principles of *conservation of mass and momentum* to determine flow. There are three available methods for routing flow in SWMM 5.0. These methods increase in their level of complexity and sophistication and warrant discussion.

The Steady State Routing Method. This method represents the simplest type of routing possible by assuming that within each computational time step, flow is uniform and steady. Thus, it simply translates inflow hydrographs at the upstream end of the conduit to the downstream end, with no delay or change in shape. The Manning's equation is used to relate flow rate to flow area (or depth). This type of routing cannot account for channel storage, backwater effects, entrance/exit losses, flow reversal or pressurized flow. It can only be used with dendritic conveyance networks, where each node has only a single outflow link (unless the node is a divider in which case two outflow links are required). This form of routing is insensitive to the time step employed and is only appropriate for preliminary analysis using long-term continuous simulations.

The Kinematic Wave Routing Method. This method solves the continuity equation along with a simplified form of the momentum equation in each conduit. The latter requires that the slope of the water surface equal the slope of the conduit. The maximum flow that can be conveyed through a conduit is the full-flow value derived by the Manning's equation. Any flow in excess of this entering the inlet node is either lost from the system or can pond atop the inlet node and be re-introduced into the conduit as capacity becomes available. Kinematic wave routing allows flow and area to vary both spatially and temporally within a conduit. This can result in attenuated and delayed outflow hydrographs as inflow is routed through the channel. However, this form of routing cannot account for backwater effects, entrance/exit losses, flow reversal, or pressurized flow, and is also restricted to dendritic network layouts. It can usually maintain numerical stability with moderately large time steps, on the order of 5 to 15 minutes. If the aforementioned effects are not expected to be significant, then this alternative can be an accurate and efficient routing method, especially for long-term simulations.

The Dynamic Wave Routing Method. Dynamic Wave routing solves the complete one-dimensional Saint Venant flow equations and therefore produces the most theoretically accurate results. These equations consist of the continuity and momentum equations for conduits and a volume continuity equation at the nodes. With this form of routing it is possible to represent pressurized flow when a closed conduit becomes full, such that flows can exceed the full-flow Manning's equation value. Flooding occurs when the water depth at a node exceeds the maximum available depth, and the excess flow is either lost from the system or can pond atop the node and re-enter the drainage system. Dynamic wave routing can account for channel storage, backwater, entrance/exit losses, flow reversal, and pressurized flow. Because it couples together the solution for both water levels at nodes and flow in conduits, it can be applied to any general network layout, even those containing multiple downstream diversions and loops. It is the method of choice for systems subjected to significant backwater effects due to downstream flow restrictions and with flow regulation via weirs and orifices. This generality comes at a price of having to use much smaller time steps, on the order of a minute or less. SWMM 5.0 will automatically reduce the user-defined maximum time step as needed to maintain numerical stability.

For purposes of this study, the Kinematic Wave Routing Method was selected for use in the models since it determines when ponding will occur at a junction or when pressure flow reaches critical within the conduit. These effects provide an indication that the conveyance system capacity would have been exceeded. For study purposes, the level of sophistication to determine backwater effects, entrance/exit losses, flow reversal, or pressurized flow is not warranted. These issues are deemed not significant; therefore the kinematic wave routing is acceptable.

3.5.8 Detention/Desiltation Basins

In addition to inlets, culverts and junction structures, the City's storm drain system incorporates the use of detention/desiltation basins. The purpose of a detention basin is to store storm water runoff flows for the attenuation the peak–flow resulting from a storm event. The typical detention basin is composed of an entrance conveyance, which may or may not have a bypass structure. The basin will have a defined volume that can be contained within its footprint. The basin can be a natural depression or can be man-made of earth or concrete. The outlet of a basin can be designed to meter out the contained storm water by either a perforated riser outlet or some other flow metering device at a specified depth. In addition, the outlet must have a bypass weir in case there is more runoff than the design capacity of the basin. Detention basins can be designed as desiltation basins by providing an internal forebay, lengthening the path of travel to allow for natural deposition to occur, adding vegetation to aid in sediment deposition, and increasing the residence time of storage. By extending the containment time of runoff, the basin becomes a primary clarifier allowing suspended sediment to settle out of the storm water.

In SWMM 5.0, detention basins are represented as storage unit type nodes. Storage units can represent any system node that has a storage volume. A stage-storage diagram is used to represent the volumetric properties of the detention basin. The input parameters for storage units include invert elevation, maximum depth, depth-surface area data and evaporation potential (USEPA, 2005).

3.6 Identification of Facilities that Require Improvement

The City has requested a limited Hydrology/Hydraulic evaluation to determine the need for new facilities or improvements, verify capacity of conveyances assuming proposed development is complete, and to justify costs associated with the construction of the proposed infrastructure. To determine the need for a new facility, the proposed project must provide a measure of storm water protection by:

(1) collecting runoff from new development/construction;

(2) providing relief from localized flooding;

(3) complete or improve an existing drainage system;

(4) reducing soil erosion by controlling runoff within a conveyance corridor;

(5) identified as an integral part of the Drainage Master Plan.

To accurately depict the function of the proposed conveyance, information such as the width, depth, and cross-section were determined from available GIS data provided by the City. Field conditions, such as type of development, roughness, and street widths were based on standard acceptable values. Where slope information was not available, it was estimated from existing topography and/or aerial mapping. Selection of model parameters were based on recommended inputs and sound engineering judgment. To test the need and capacity for such a facility, parameters for the 100-year 6-hour storm event and the 25-year 6-hour storm event have been compiled, and models

have been prepared for the requested projects. A typical roadway geometry that has a 2.0% crown with a width of 40 feet and depth of 0.5 foot depth for a curb was used as one of assumptions to determine flooding limits. Details on input and output data for the hydraulic modeling can be found in Appendix A.
The projects identified below have been modeled to determine need, function, and the cost of construction.

3.6.1 Drainage Project AC - Highland Drive Drainage Project

Drainage project AC originates within an existing residential area. Its main purpose is to minimize localized flooding within the community by extending the current facilities and providing improvements for discharge from the existing basin. The drainage improvements are comprised of three main components as follows:

- upstream extension of a 36-inch RCP;
- downstream construction of a concrete trapezoidal channel; and
- downstream parallel 18-inch discharge culvert to improve outlet capacity.

The contributing drainage area of 50.0 acres generates a peak discharge volume of Q=64.1 cfs for the 100-year, 6-hour storm event. This calculated runoff volume exceeds the assumed roadway geometry of 0.5 foot depth and creates localized flooding within the traveled way. Therefore, it is recommended to construct the 36-inch RCP. Due to the volume and velocity of runoff downstream, it is recommended to construct the trapezoidal channel to minimize erosion leading to the downstream basin. In addition, to prevent overtopping of the banks at the basin, it is also recommended to construct an 18-inch culvert in parallel to the existing 18-inch culvert to provide more conveyance capacity for the basin.

3.6.2 Drainage Project BB-1 - Washington Street Drainage Improvements-Phase I

Drainage project BB-1 originates within an existing residential area. Its main purpose is to minimize localized flooding within the community by providing a new facility and providing an outlet for the existing facilities. The drainage improvement for Project BB-1 is comprised of an 18-inch RCP that needs to tie into an existing 72-inch RCP trunk line.

A contributing drainage area of 26.0 acres generates a peak discharge volume of Q=37.3 cfs for the 100-year, 6-hour storm event. This calculated runoff volume exceeds the assumed roadway geometry flood limits of 0.5 foot depth and can create localized flooding at the existing facilities of Junction J2, J5, and J8. The flooding can be minimized by providing conveyance capacity with the proposed 18-inch RCP. Therefore, it is recommended to construct the proposed 18-inch RCP. The proposed culvert will connect to an existing junction structure for a 72-inch RCP trunk line on the east side of the railroad tracks. Upon checking the capacity of the existing 72-inch RCP trunk line, it is noted that this existing facility flows under pressure and is subject to flooding for the 100-year, 6-hour storm event. This creates a backwater effect on the proposed 18-inch RCP. Typical rainfall simulations at the 25-year, 6-hour storm event do not show flooding.

3.6.3 Drainage Project BB-2 - Washington Street Drainage Improvements-Phase II

Drainage project BB-2 originates within an existing residential area. Its main purpose is to minimize localized flooding within the community by providing a new facility and providing an outlet for the existing facilities. The drainage improvement for Project BB-2 is comprised of a 36-inch RCP that needs to tie into an existing 84-inch RCP trunk line.

The contributing drainage area of 34.1 acres generates a peak discharge volume of Q=33.4 cfs for the 100-year, 6-hour storm event. This calculated runoff volume exceeds the assumed roadway geometry flood limits of 0.5 foot depth and can create localized flooding at the existing facilities of Junction J12, and J22. The flooding can be minimized by providing conveyance capacity with the proposed 36-inch RCP. Therefore, it is recommended to construct the proposed 36-inch RCP. The proposed culvert will connect to an existing junction structure for an 84-inch RCP trunk line on the east side of the railroad tracks. Upon checking the capacity of the existing 84-inch RCP trunk line, it is noted that this existing facility flows under pressure and is subject to flooding for the 100-year, 6-hour storm event. This creates a backwater effect on the proposed 36-inch RCP. Typical rainfall simulations at the 25-year, 6-hour storm event do not show flooding.

3.6.4 Drainage Project BCA - Park Drive/Tamarack Avenue Drainage Project

Drainage project BCA originates within an existing residential area. Its main purpose is to minimize localized flooding within the community by providing new drainage improvements to accommodate stormwater runoff. The drainage improvement is comprised of a new 24-inch RCP.

The contributing drainage area of 40.6 acres generates a peak discharge volume of Q=32.8 cfs for the 100-year, 6-hour storm event. This calculated runoff volume would exceed the assumed roadway geometry flood limits of 0.5 foot depth and would create localized flooding within the traveled way, downstream of Junction J8 and J9. The localized flooding within the City street can be minimized by providing conveyance capacity with the proposed 24-inch RCP. Therefore, it is recommended to construct the proposed 24-inch RCP and a junction structure to connect the improvements to an existing 48-inch drainage facility.

3.6.5 Project BCB – Magnolia Avenue Project

Drainage project BCB originates within an existing residential area. Its main purpose is to minimize localized flooding within the community by providing new drainage improvements to accommodate stormwater runoff. The drainage improvement is comprised of a new 30-inch RCP.

The contributing 19.6 acres of drainage area generates a peak discharge volume of Q=20.6 cfs for the 100-year, 6-hour storm event. This calculated runoff volume would not exceed the assumed roadway geometry flood limits of 0.5 foot depth. However, it can create localized flooding within the traveled way, downstream of Junction J3. The localized flooding within the City street can be minimized by providing conveyance capacity with the proposed 30-inch RCP. It is recommended to construct the 30-inch RCP and a junction structure to connect to an existing 48-inch drainage facility.

3.6.6 Drainage Project BCC - Chestnut Avenue Drainage Project

Drainage project BCC originates within an existing residential area. Its main purpose is to minimize localized flooding around Chestnut Street adjacent to Carlsbad High school and to accommodate stormwater runoff within the community by providing a drainage improvement comprised of a new 36-inch RCP.

The contributing drainage area of 84.8 acres generates a peak discharge volume of Q=82.9 cfs for the 100-year, 6-hour storm event. This calculated runoff volume would exceed the assumed roadway geometry flood limits of 0.5 foot depth and would create localized flooding within the traveled way, downstream of Junction J14, J15 and J16. The localized flooding within the City street can be minimized by providing conveyance capacity with the proposed 36-inch RCP. Therefore, it is recommended to construct the proposed 36-inch RCP. The proposed culvert will connect to an existing junction structure for an existing 42-inch lateral further downstream on Chestnut Street.

3.6.7 Drainage Project C1 - Carlsbad Boulevard South Drainage Improvements

Drainage project C1 is a proposed 12-foot by 5-foot Reinforced Concrete Box culvert with a length of 50 linear feet (L.F.). Its main purpose is to provide additional capacity to the existing bridge that conveys the Encinas Creek flow beneath the southbound lanes of Carlsbad Boulevard.

The contributing drainage area of 2,300 acres generates a peak discharge volume of Q=1,396.0 cfs for the 100-year, 6-hour storm event. When existing conditions are modeled (two 10-foot by 5-foot RCB) there is evidence of flooding at junction J83. The addition of a 12-foot by 5-foot Reinforced Concrete Box minimizes the flooding and stabilizes the velocity of flow. It is recommended to construct the additional 12-foot by 5-foot Reinforced Concrete Box culvert to provide for additional capacity to the existing bridge.

3.6.8 Drainage Project C2 - Paseo Del Norte Drainage Improvements

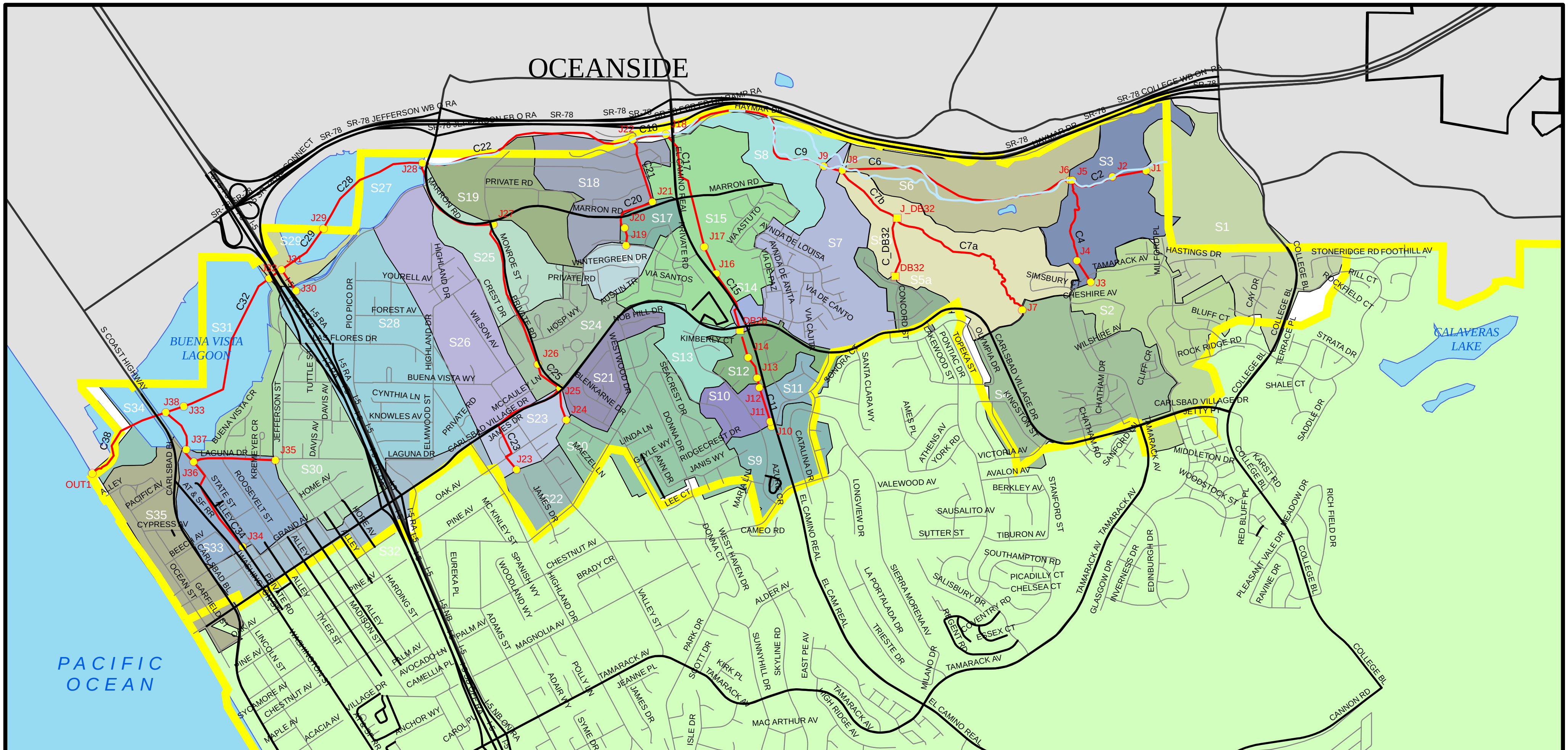
Drainage project C2 is a proposed 10-foot by 4-foot Reinforced Concrete Box culvert with a length of 90 linear feet (L.F.). Its main purpose is to provide additional capacity to the existing bridge that conveys the Encinas Creek flow beneath the lanes of Paseo Del Norte.

The contributing drainage area of 1,993 acres generates a peak discharge volume of Q=1184.0 cfs for the 100-year, 6-hour storm event. When existing conditions are modeled (three 10-foot by 5-foot RCB) there is evidence of flooding at junction J70. The addition of a 10-foot by 5-foot Reinforced Concrete Box minimizes the flooding and stabilizes the velocity of flow. It is recommended to construct the additional 10-foot by 5-foot Reinforced Concrete Box culvert to provide for additional capacity to the existing bridge.

3.6.9 Drainage Project DH - Altiva Place Canyon Restoration and Enhancement Project

Drainage project DH originates within an existing residential area. Its main purpose is to reduce erosive velocities within the conveyance channel and aid in the reduction of sediment. The drainage improvement is comprised of 800 feet of unlined trapezoidal channel.

The contributing drainage area of 358 acres generates a peak discharge volume of Q=628.0 cfs for the 100-year, 6-hour storm event. When existing conditions are modeled as a trapezoidal channel, there is evidence of high velocities and flooding at the connection to the existing 72-inch lateral for a short period of time. The existing channel must connect to an existing 72-inch lateral under Alicante Road. The typical 25-year, 6-hour storm event does not show evidence of flooding through the modeled alignment. It is recommended to install the channel enhancement to minimize the runoff velocities and promote efficient conveyance to the 72-inch RCP.



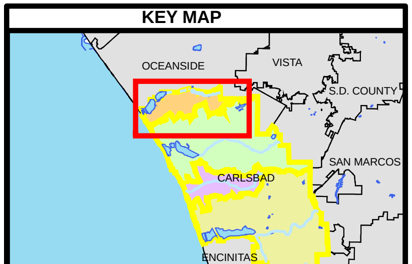
LEGEND

- CREEKS
- BODY OF WATER
- AREA NOT INCLUDED IN DRAINAGE BASIN
- SWMM JUNCTIONS
- SEDIMENTATION BASINS
- SWMM FLOWLINES

DRAINAGE BASINS

- A
- B
- C
- D

KEY MAP



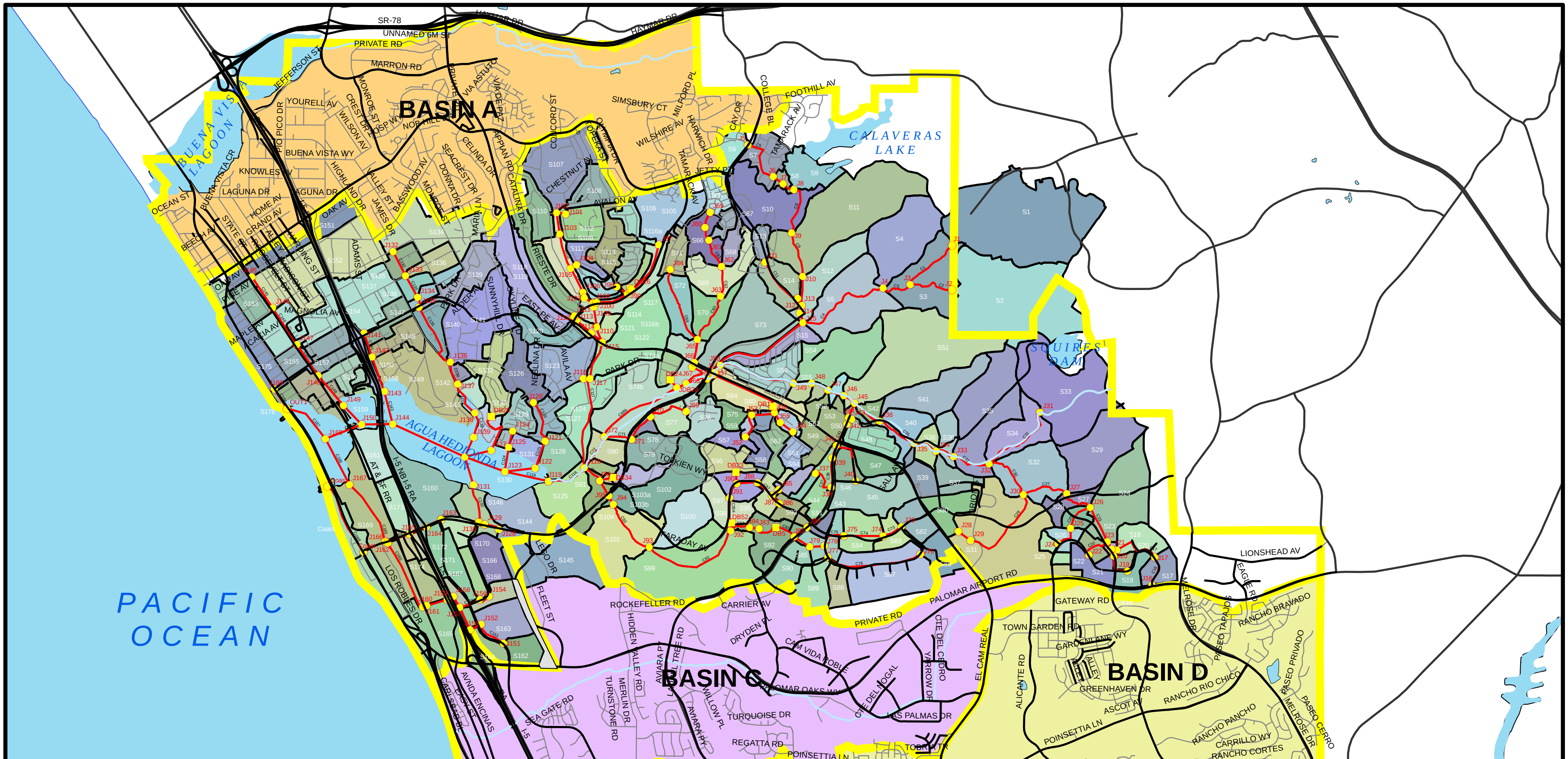
N
W E
S

0 1,250 2,500 5,000
Feet

FIGURE 3 - 1
BASIN A - SWMM MODEL WATERSHED AREA
BUENA VISTA CREEK

PROJECT LOCATION	DATE	PROJECT NUMBER
CARLSBAD, CALIFORNIA	FEB 2007	128290

BROWN AND CALDWELL
SAN DIEGO, CALIFORNIA



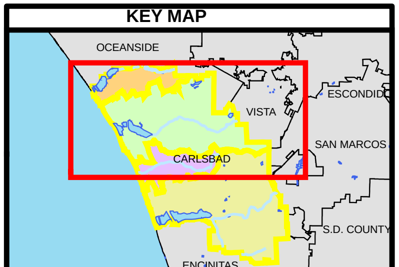
LEGEND

- CREEKS
- BODY OF WATER
- AREA NOT INCLUDED IN DRAINAGE BASIN
- SWMM JUNCTIONS
- SEDIMENTATION BASINS
- SWMM FLOWLINES

DRAINAGE BASINS

- A
- B
- C
- D

KEY MAP



N
W E
S

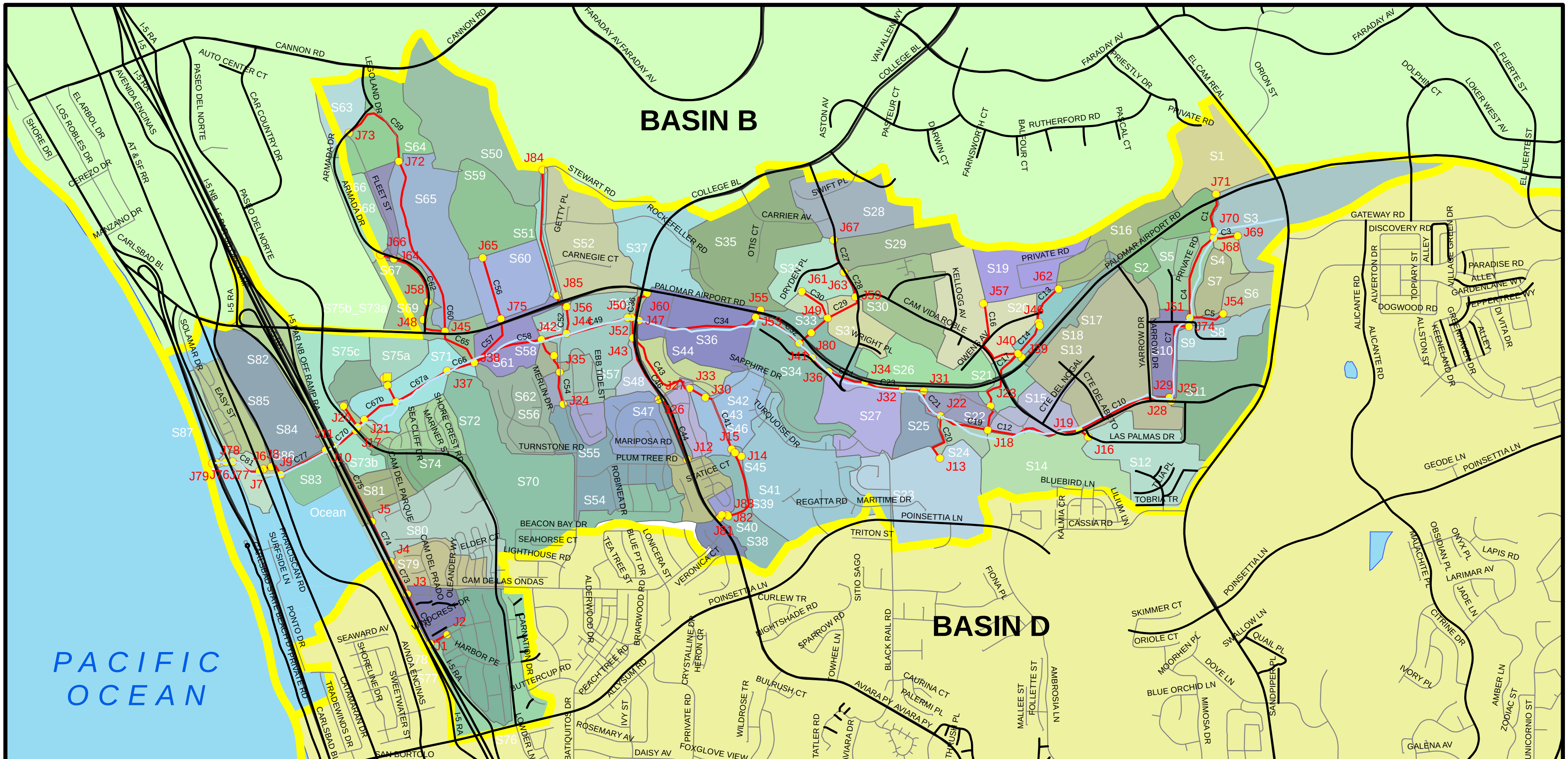
0 2,500 5,000 10,000 Feet

FIGURE 3 - 2

BASIN B - SWMM MODEL WATERSHED AREA
AGUA HEDIONDA CREEK

PROJECT LOCATION	DATE	PROJECT NUMBER
CARLSBAD, CALIFORNIA	FEB 2007	128290

BROWN AND CALDWELL
SAN DIEGO, CALIFORNIA



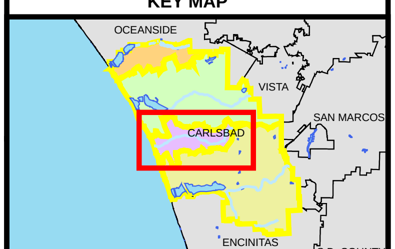
LEGEND

- CREEKS
- BODY OF WATER
- AREA NOT INCLUDED IN DRAINAGE BASIN
- SWMM JUNCTIONS
- SEDIMENTATION BASINS
- SWMM FLOWLINES

DRAINAGE BASINS

- A
- B
- C
- D

KEY MAP



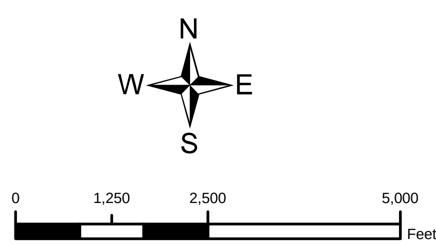
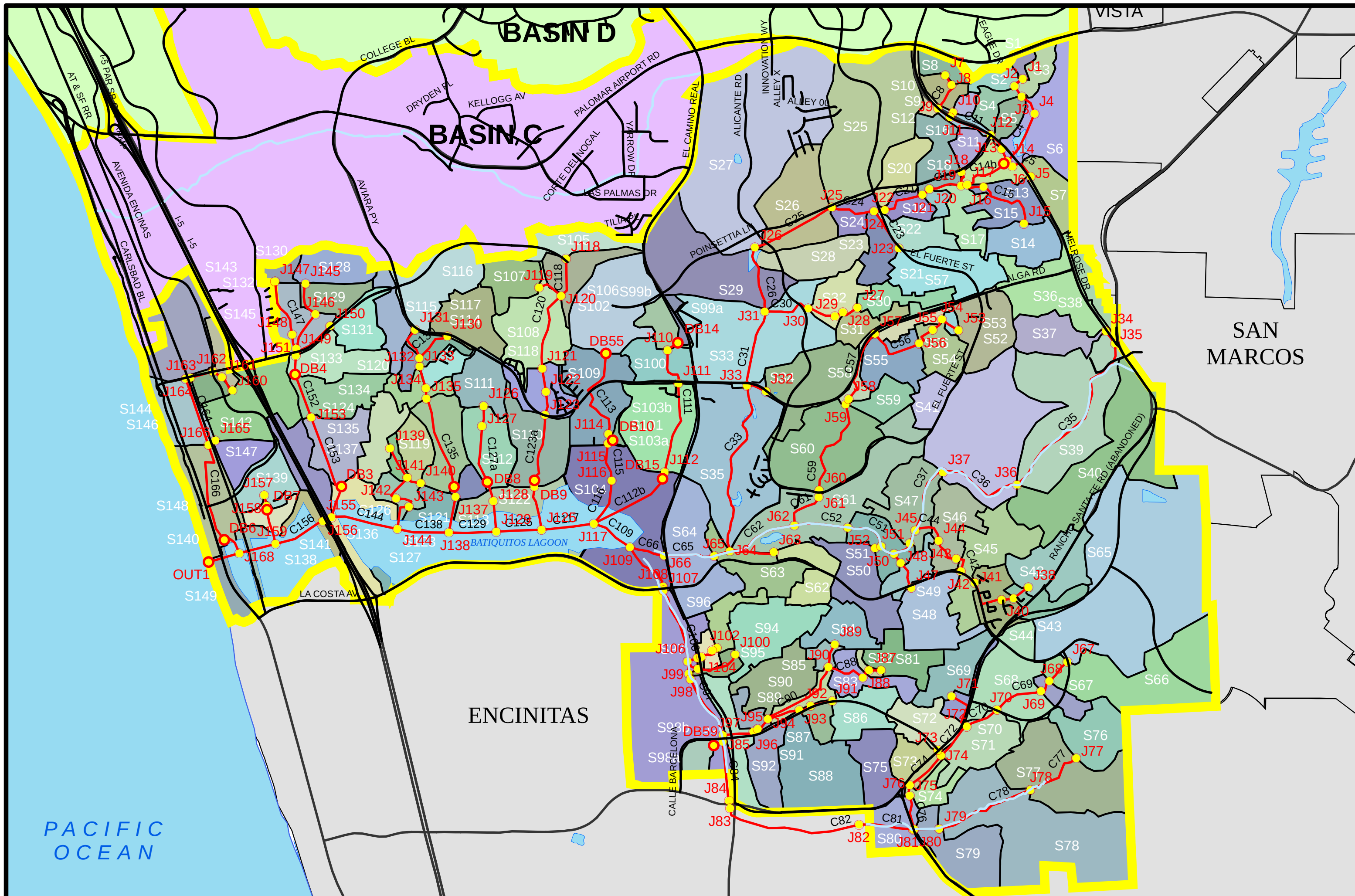


FIGURE 3-3

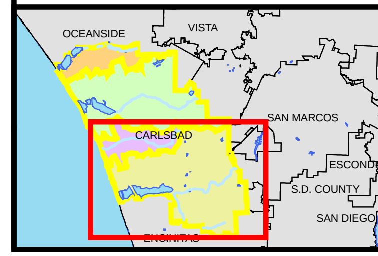
BASIN C - SWMM MODEL WATERSHED AREA ENCINAS CREEK

PROJECT LOCATION	DATE	PROJECT NUMBER
CARLSBAD, CALIFORNIA	FEB 2007	128290

BROWN AND CALDWELL
SAN DIEGO, CALIFORNIA



KEY MAP



LEGEND

- CREEKS
- BODY OF WATER
- AREA NOT INCLUDED IN DRAINAGE BASIN
- SWMM JUNCTIONS
- SEDIMENTATION BASINS
- SWMM FLOWLINES

DRAINAGE BASINS

- A
- B
- C
- D

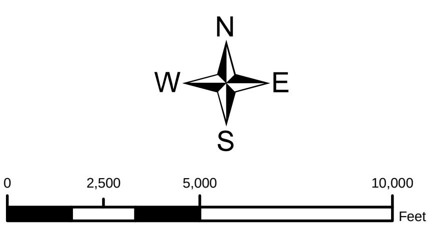


FIGURE 3 - 4

BASIN D - SWMM MODEL WATERSHED AREA BATIQUITOS/ SAN MARCOS CREEK

PROJECT LOCATION	DATE	PROJECT NUMBER
CARLSBAD, CALIFORNIA	FEB 2007	128290

BROWN AND CALDWELL

SAN DIEGO, CALIFORNIA